SECTION 5: CONCRETE STRUCTURES

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5.1 SCOPE

This Section contains guidelines to supplement provisions of Section 5, Concrete Structures, of the AASHTO LRFD Bridge Design Specifications. These provisions apply to the design of bridges, retaining walls, and other appurtenant highway structure components constructed of normal density concrete reinforced with steel bars, welded wire reinforcement, prestressing strands, prestressing bars, or prestressing wires. Concrete deck design criteria are specified in Section 9 of these guidelines.

All design engineers are advised to review the example problems in Appendix – A of these guidelines for proper and correct application of various provisions of the AASHTO LRFD Specifications and these guidelines for design of bridge structural components.

5.4 MATERIAL PROPERTIES

5.4.1 General

Design should be based on the material properties cited in these guidelines. The contract documents shall specify the grades or properties of all materials to be used and shall be in conformance with the latest edition of the ADOT Standard Specifications for Road and Bridge Construction. All structural concrete shall meet or exceed ADOT Class S requirements unless noted otherwise.

5.4.2 Normal and Structural Lightweight Concrete

5.4.2.1 Compressive strength:

For non-prestressed applications, concrete strength greater than 5 ksi will require ADOT Materials and Bridge Groups approval. Lightweight concrete shall not be used as a structural material. Normal weight non-prestressed concrete shall have minimum strengths, f'_c, at 28 days as follows:

Components	f'_c (ksi)
Decks except barriers	4.5
Bridge concrete barriers	4.0
Substructures (abutments, piers, foundations and Drilled Shafts)	3.5
All other Class ‘S’ Concrete	3.0

Prestressed Precast Concrete shall have initial and final concrete strengths as specified in the table below. Concrete strengths greater than shown in that table may be used when required by design and approved by ADOT Materials and Bridge Groups.

	Initial	Final
Minimum	f'_{ci} = 4.0 ksi	f'_c = 5.0 ksi
Maximum	f'_{ci} = 5.0 ksi	f'_c = 6.5 ksi

Concrete for cast-in-place post-tensioned box girder bridges shall have initial and final concrete strengths as specified in the table below. Concrete strengths greater than the maximum limit may only be used when required by design and approved by ADOT Materials and Bridge Groups.

	Initial	Final
Minimum	f'_{ci} = 3.5 ksi	f'_c = 4.5 ksi
Maximum	-	f'_c = 5.0 ksi

Final maximum compressive stress, f'_c, of up to 6.0 ksi for cast-in-place post-tensioned box girder bridges in the Phoenix and Tucson metropolitan areas' projects may be specified.

5.4.2.3 Shrinkage and Creep

Shrinkage and creep of concrete, relaxation of prestressing steel, and shrinkage of deck composite cross section are time dependent events.

Precast Prestressed Application:
Approximate Lump Sum Estimate of Time-Dependent Losses described in the current edition of AASHTO LRFD Article 5.9.5.3, may be used for all precast pretensioned conventional applications (spans not to exceed 140'-0"). For unusual structures or structure spans over 140'-0", ADOT Bridge Group shall be consulted regarding the loss calculation method to be used.

Post-Tensioned Application:
In lieu of a more detailed analysis, prestress losses in post-tensioned members, constructed and prestressed in a single stage shall be based on AASHTO LRFD Article 5.9.5, third edition, 2004, the details of which are reproduced in Article 5.9.5 of these guidelines and are demonstrated in the examples in Appendix A for convenience.

5.4.2.4 Modulus of Elasticity

Based on an assumed unit weight of concrete of w_c = 0.145 kcf, the modulus of elasticity, E_c, in ksi, shall be assumed to be:

$$E_c = 1820\sqrt{f'_c}\ \text{(ksi)}$$

where:

f'_c = the specified compressive strength of concrete (ksi)

For dead load calculations, the unit weight of structural concrete shall be w_c = 0.150 kcf.

5.4.3 Reinforcing Steel

All reinforcing steel shall be supplied as Grade 60 and shall be deformed bars conforming to ASTM A 615 / A 615M-96a, except for smooth wire spiral ties. Spiral ties shall be cold drawn wire conforming to ASTM A 82 (AASHTO M-32).

Welded wire fabric reinforcing shall only be used in slope paving and prefabricated panels used for sound walls. Welded wire reinforcing shall be deformed. Deformed welded steel wire and fabric shall conform to ASTM A 496 and ASTM A 497. For structural applications in places other than slope paving or prefabricated panels, only deformed steel bars conforming to ASTM 706 shall be used if approved by ADOT Bridge Group. Where ductility is to be assured or where welding is required, steel conforming to the requirements of ASTM A706 shall be specified.

All new bridge construction located above an elevation of 4000 feet, or for areas where de-icing chemicals are used, deck slabs, barriers, anchor slabs and approach slabs reinforcing as well as portions of reinforcement projecting into the deck slabs shall be epoxy coated. Reinforcing bars shall conform to the requirements of ASTM A 775 or ASTM A 934. Welded wire fabric reinforcing shall conform to ASTM A 884, Class A.

5.4.4 Prestressing Steel

Prestressing steel for precast prestressed members and cast-in-place post-tensioned members shall be low-relaxation type and have the properties as defined in AASHTO LRFD Article 5.4.4.1 and AASHTO LRFD Table 5.4.4.1-1. For most applications, 0.5-inch diameter strands should be specified for prestressed members. For cast-in-place post-tensioned bridge girders 0.6-inch diameter strands may be used.

The yield point stress of prestressing steel, f_{py}, shall be equal to 0.90 f_{pu} for low relaxation strands, as shown in AASHTO LRFD Table 5.4.4.1-1.

5.5.4.2 Resistance Factors

For tensioned-controlled sections of post-tensioned box girder bridges, the resistance factor shall be taken as $\varphi = 0.95$. AASHTO LRFD Article 5.5.4.2 shall be used for all other cases.

5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

5.7.2 Assumptions for Strength and Extreme Event Limit States

5.7.2.1 General

Unbonded tendons shall not be used.

5.7.3 Flexural Members

Unbonded tendons shall not be used.

5.7.3.2 Flexural Resistance

For post-tensioned box girders, in determining the negative flexural resistance, the temperature and shrinkage reinforcement or the distribution reinforcement may be used at the design engineer's discretion. In determining the positive flexural resistance, the longitudinal flange reinforcing may be used, if necessary, to reduce reinforcement congestion.

5.7.3.4 Control of Cracking by Distribution of Reinforcement

For structures other than deck slabs, the tensile stress in the mild steel reinforcement at the service limit state, f_s, shall be limited to $0.6f_y$. When the Strut-and-Tie model is used for design of structures and components, crack control provisions specified in AASHTO LRFD Article 5.6.3.6 shall be met.

5.7.3.6 Deformations

5.7.3.6.1 General

Deflections shall be limited to the values shown in the following table:

Loads Carried by the Bridge	Maximum Deflection
Vehicular Load	Span / 800
Vehicular and Pedestrian Loads	Span / 1000
Pedestrian Load	Span / 1000
Vehicular Load on Cantilever Arms	Span / 300
Vehicular and Pedestrian Loads on Cantilever Arms	Span / 375
Pedestrian Load on Cantilever Arms	Span / 375

The design engineer may determine the deflections due to creep and shrinkage based on the next article of these guidelines (5.7.3.6.2). Creep and shrinkage values presented in AASHTO LRFD Article 5.4.2.3 may also be used for more precise determination of long-term deflections as suggested in AASHTO LRFD Commentary 5.7.3.6.1. If the method specified in AASHTO LRFD Article 5.4.2.3 is used, the calculation should include construction stage analysis considering transition of section properties, maturity of concrete, prestress losses, and change of structural model, that is, transition from simple to continuous span.

5.7.3.6.2 Deflection and Camber

Post-tensioned Box Girder:

The instantaneous deflection shall be calculated using the dead load including barriers but not future wearing surface, a modulus of elasticity, $E_c = 1820\sqrt{f'_c}$ ksi, gross sectional properties, and calculated final losses.

The long-term deflection shall be calculated according to AASHTO LRFD Article 5.7.3.6.2 using the gross moment of inertia, I_g.

The total deflection may be obtained by multiplying the instantaneous deflection by a factor of three if compression steel is not included in moment of inertia calculations. For simple span bridges, an additional parabolic shaped deflection with a peak equal to 3/8 inch per 100 feet should be added to the total deflection.

Camber and screed elevations shown on the plans shall be based on the final long-term deflection. Continuous bridges may exhibit negative camber in some spans under various conditions. When this happens, the long-term deflection shall be set to for those spans exhibiting negative camber only.

Precast prestressed I-girder:

Deflection calculations for precast prestressed I-girders shall be based on a cumulative construction stage analysis considering transition of section properties, maturity of concrete, prestress losses, and transition of the structure from simple to continuous. Construction stages should be defined realistically and the factors mentioned above should be applied appropriately according to the conditions for each construction stage. Design engineers are advised to review Appendix – A, Example problems No. 3 and 4 for deflection calculation details.

The release, initial and final deflections shall be shown on the plan sheets. Deflections shall be shown in thousandths of a foot at the tenth points of each span.

The release deflection equals the deflection the prestress girder undergoes at the time of release of strands. This includes the deflection due to dead load of the girder, elastic shortening, and relaxation of steel at transfer, as specified in AASHTO LRFD Article 5.9.5.3, third edition, 2004. The release deflection values shall be shown in the contract plans.

The initial deflection includes the deflection due to the dead load of the girder, the initial prestressing and the effects of creep and shrinkage up to the time of erection and prior to the diaphragm or deck pours. The time of erection should be assumed to be 60 days after release.

The total deflection includes the deflection due to the dead load of the deck, diaphragms, barriers, and the effects of long term creep on the composite girders. The future wearing surface shall be excluded from deflection calculations.

Minimum build-up at the edge of Type III girder and smaller shall be of ½ inch. For Type IV, V and VI girders the minimum build-up shall be 1 inch. This minimum build-up at the critical section will ensure that the flange of the girder will not encroach into the gross depth of the slab.

The top of the erected girders shall be surveyed in the field prior to placement of the deck forming. Encroachment into the slab of up to ½ inch will be allowed for random occurrences.

Design engineers should provide a note on the plans stating that the bridge seat elevations shall be verified by the contractor prior to the erection of the girders.

5.8 SHEAR AND TORSION

5.8.2 General Requirements

5.8.2.6 Types of Transverse Reinforcement

Welded wire fabric and anchored prestressed tendons shall not be used as transverse shear reinforcement without approval from ADOT Bridge Group.

5.8.3 Sectional Design Model

Note that N_u in AASHTO LRFD Articles 5.8.3.4.2 and 5.8.3.5 is not the prestress force but is the externally applied factored axial force, taken positive if tensile.

5.8.3.5 Longitudinal Reinforcement

AASHTO LRFD Equation 5.8.3.5-1 shall be evaluated at each section where simply supported girders are made continuous for live loads or where longitudinal reinforcement is discontinuous.

AASHTO LRFD Equation 5.8.3.5-2 shall be evaluated at the inside edge of the bearing area of simple end supports to the section of critical shear (d_v from the internal face of support, AASHTO LRFD Article 5.8.3.2). The values of V_u, V_s, V_p, and θ, calculated for the section d_v from the face of the support, may be used.

5.8.4 Interface Shear Transfer – Shear Friction

For precast prestressed members, only the composite dead load and live load shall be considered when calculating interface shear. For I-Girders, the following note shall be shown on the plans: The contact surface of the top flange shall be roughened to a depth of approximately ¼ inch. Interface shear shall not be considered in the design of post-tensioned bridge members.

5.9 PRESTRESSING AND PARTIAL PRESTRESSING

5.9.1 General Design Considerations

5.9.1.4 Section Properties

Section properties shall be based on gross area of members for cast-in-place post-tensioned members. Section properties shall be based on transformed area of bonded prestressing strand for precast prestressed members. Gross section properties shall be used for deflection calculations and live load distribution for precast prestressed members.

5.9.3 Stress Limitations for Prestressing Tendons

For post-tensioned members, overstressing for short period of time to offset seating and friction losses is permitted but the maximum allowable jacking stress for low-relaxation strands shall be limited to $0.78f_{pu}$.

For precast prestressed members, the maximum allowable jacking stress for low relaxation strands shall be limited to $0.75f_{pu}$. Overstressing the prestressing steel to offset seating losses is not permitted.

5.9.4 Stress Limits for Concrete

		Load Cases				
		Before Time-Dependent Losses	**After Losses**			
			DC + Prestress	**Service Limit I**	**Service Limit III**	**0.5(DW+DC+ Prestress) + (LL + IM)**
Compression (ksi)		$0.6f'_{ci}$	$0.45f'_c$	$0.6\phi_w f'_c$	N/A	$0.4f'_c$
Tension (ksi)	Any region of a prestressed component in which prestressing causes compressive stresses and service load effects cause tensile stresses	N/A	0 for post-tensioned boxes N/A for precast prestressed members	N/A	$0.0948\sqrt{f'_c}$ (For post-tensioned structures built on falsework, this value shall be zero. No tension shall be allowed)	N/A
	Other Regions	$0.0948\sqrt{f'_{ci}}$ $\leq 0.2ksi$	N/A	N/A	N/A	N/A

5.9.5 Loss of Prestress

In lieu of a more detailed analysis for determining prestress losses in post-tensioned members constructed and prestressed in a single stage, relative to the stress immediately before transfer, the AASHTO LRFD Specifications, third edition, 2004 method may be used. The method is reiterated below for convenience.

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR\,2}$$

where:

Δf_{pT} = total loss (ksi)

Δf_{pF} = loss due to friction (ksi)

Δf_{pA} = loss due to anchorage set (ksi)

Δf_{pES} = loss due to elastic shortening (ksi)

Δf_{pSR} = loss due to shrinkage (ksi)

Δf_{pCR} = loss due to creep of concrete (ksi)

$\Delta f_{pR\,2}$ = loss due to relaxation of steel after transfer (ksi)

Also, consideration should be given to a loss of tendon force, as indicated by pressure readings, within the stressing equipment.

Loss Due to Friction

For multi-span bridges, the cable path should have its low point at or near mid span. Design should be based on usage of galvanized rigid ducts with K = 0.0002 and μ = 0.25. However, μ value of 0.15 may be used when comparing initial force coefficient with values shown in the “Friction Coefficients for Post-Tensioning Tendons” table below.

Losses due to friction between the internal prestressing tendons and the duct will be taken as:

$$\Delta f_{pF} = f_{pj}\,(1 - e^{-(Kx + \mu\alpha)}) \qquad (5.9.5.2.2b\text{-}1)$$

where:

f_{pj} = stress in the prestressing steel at jacking (ksi)
x = length of a prestressing tendon from the jacking end to any point under consideration (ft)
K = 0.0002, wobble friction coefficient (per ft of tendon)
μ = coefficient of friction

α = sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation (rad)
e = base of Napierian logarithms

Value of μ should be based on experimental data for the materials specified and shall be within the values given in the "Friction Coefficients for Post-Tensioning Tendons" table below. In the absence of such data, a value of $\mu = 0.25$ may be used.

For tendons confined to a vertical plane, α shall be taken as the sum of the absolute values of angular changes over length x.

For tendons curved in three dimensions, the total three-dimensional angular change α shall be obtained by vectorially adding the total vertical angular change, α_v and the total horizontal angular change, α_h.

Friction Coefficients for Post-Tensioning Tendons

Type of Steel	Type of Duct	K	μ
Wire or Strand	Rigid and semi-rigid Galvanized metal sheathing	0.0002	0.15-0.25
	Polyethylene	0.0002	0.23
High-strength bars	Galvanized metal sheathing	0.0002	0.30

Loss Due to Anchorage Set

$$\Delta f_{pA} = \frac{2\Delta f_{pF} X}{L}$$

where:

$$X = \sqrt{\frac{E_p(\Delta L)L}{12\Delta f_{pF}}}$$

where:

X = anchor set length, ft
L = span length, ft
ΔL = 3/8 inch (anchor set)

Loss Due to Elastic Shortening

In calculating the loss due to elastic shortening, Δf_{pES}, in post-tensioned members other than slab systems, the following corrected equation shall be used:

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt}\left(I_g + A_g e_m^2\right) - e_m M_g A_g}{A_{ps}\left(I_g + A_g e_m^2\right) + \frac{A_g \cdot I_g \cdot E_{ci}}{E_p}\left(\frac{2N}{(N-1)}\right)} \qquad \text{(C5.9.5.2.3b-1)}$$

where:

A_{ps} = area of prestressing steel (in^2)
A_g = gross area of section (in^2)
E_{ci} = modulus of elasticity of concrete at transfer (ksi)
E_p = modulus of elasticity of prestressing steel (ksi)
e_m = average eccentricity at mid span (in)
f_{pbt} = stress in prestressing steel immediately prior to transfer as specified in AASHTO LRFD Table 5.9.3-1, (ksi)
I_g = moment of inertia of gross concrete section (in^4)
M_g = midspan moment due to member self weight (kip-in)
N = number of identical prestressing tendons

The value of Δf_{pES} should be determined at the center section of the span for simple spans and at critical sections for continuous spans.

For slab systems, the value of Δf_{pES} may be taken as 25 per cent of that obtained from Equation C5.9.5.2.3b-1, as shown above.

For post-tensioned construction, the value of Δf_{pES} may be further reduced below those implied by Equation C5.9.5.2.3b-1, as shown above, with proper tensioning procedures such as stage stressing and retensioning.

Loss Due to Shrinkage

Loss of prestress for post-tensioned members, in ksi, due to shrinkage may be taken as:

$$\Delta f_{pSR} = (13.5 - 0.123H)$$

where:

H = the average annual ambient relative humidity (percent), may be taken as 40%.

Loss Due to Creep of Concrete

Prestress loss, in ksi, due to creep may be taken as:

$$\Delta f_{pCR} = 12.0 f_{cgp} - 7.0\ \Delta f_{cdp} \geq 0$$

where:

f_{cgp} = concrete stress at center of gravity of prestressing steel at transfer (ksi)
Δf_{cdp} = change in concrete stress at center of gravity of prestressing steel due to permanent loads, with the exception of the load acting at the time the prestressing force is applied.

Values of Δf_{cdp} should be calculated at the same section or at sections for which f_{cgp} is calculated.

Loss Due to Relaxation of Steel after Transfer

For post-tensioning with stress-relieved strands:

$$\Delta f_{pR2} = 20.0 - 0.3\Delta f_{pF} - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})$$

where:

Δf_{pF} = the friction loss below the level of 0.70 f_{pu} at the point under consideration as computed above (ksi)
Δf_{pES} = loss due to elastic shortening (ksi)
Δf_{pSR} = loss due to shrinkage (ksi)
Δf_{pCR} = loss due to creep of concrete (ksi)

For prestressing steels with low relaxation properties conforming to ASTM A 416 or E 328 (AASHTO M 203) use 30 percent of Δf_{pR2}.

For post-tensioning with 145 to 160 ksi bars, loss due to relaxation should be based on approved test data. If test data are not available, the loss may be assumed to be 3.0 ksi.

For post-tensioned members with:

Spans not greater than 250 ft,
Normal weight concrete, and
Strength in excess of 3.50 ksi at the time of prestress,

Values of creep, shrinkage, and relaxation related losses, may be determined in accordance with the provision of AASHTO LRFD Article 5.4.2.3 or this article.

For segmental construction, for all considerations other than preliminary design, prestress losses shall be determined as specified in Article 5.9.5 of these guidelines, including consideration of the time-dependent construction method and schedule shown in the contract documents.

For precast pre-tensioned members other than slab systems, in calculating the loss of prestress due to elastic shortening, Δf_{pES}, design engineers shall use AASHTO LRFD equation C5.9.5.2.3a-1 of Article 5.9.2.3.

5.9.5.3 Approximate Lump Sum Estimate of Time-Dependent Losses

Approximate lump sum estimate of time-dependent losses described in the latest edition of AASHTO LRFD Article 5.9.5.3 shall be used to calculate long term loss of prestress forces due to shrinkage and creep of concrete, and relaxation of prestressing steel for all precast pretensioned conventional bridge design.

For unusual structures and structures with spans greater than 140 feet, ADOT Bridge Group shall be consulted for the appropriate method of loss calculation to be used.

The average annual ambient relative humidity, H, shall be 40% for calculation of concrete creep and shrinkage coefficients whenever applying AASHTO LRFD Article 5.4.2.3. H shall also be 40% whenever applying both approximate and refined estimates of prestress loss calculations in AASHTO LRFD Articles 5.9.5.3 and 5.9.5.4 respectively.

Relaxation before Transfer
For precast prestressed concrete girders, the release time, which is the estimated time from stressing to transfer, may be assumed to be 36 hours. For precast prestressed girders, the initial and final losses shall include the release losses Δf_{pR1}. AASHTO LRFD Equation (5.9.5.1-1) should be modified as follows:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} + \Delta f_{pR1}$$

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$

where:

Δf_{pR1} = relaxation loss of prestress before transfer (ksi)
t = time estimated in days from stressing to transfer
f_{pj} = initial stress in the tendon at the end of stressing (ksi)
f_{py} = specified yield strength of prestressing steel (ksi)

5.9.5.4 Refined Estimates for Time-Dependent Losses

Creep
For creep of concrete, the variable, f_{cgp} defined in AASHTO LRFD Article 5.9.5.4.3b, should be calculated using the total dead load applied after prestressing, including the 25 psf future wearing surface.

5.10 Details of Reinforcement

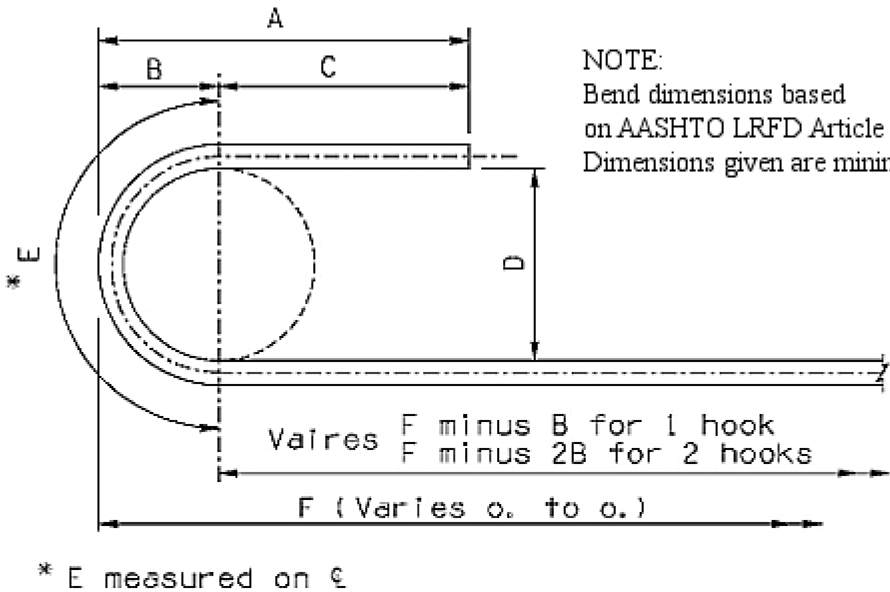
Please note that AASHTO LRFD specifications refer to main reinforcing as longitudinal reinforcing. All other reinforcing such as spirals, ties and stirrups are referred to as transverse reinforcing.

5.10.2 Hooks and Bends

5.10.2.1 Standard Hooks

The following tables shall be used.

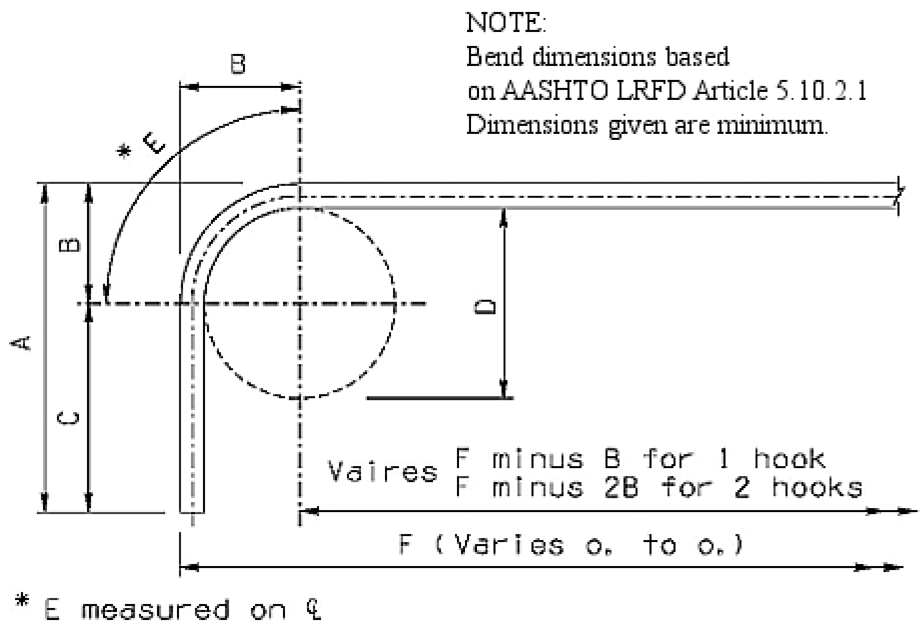
180° STD. Hook Dimensions for Longitudinal Reinforcement



NOTE:
Bend dimensions based on AASHTO LRFD Article 5.10.2.1. Dimensions given are minimum.

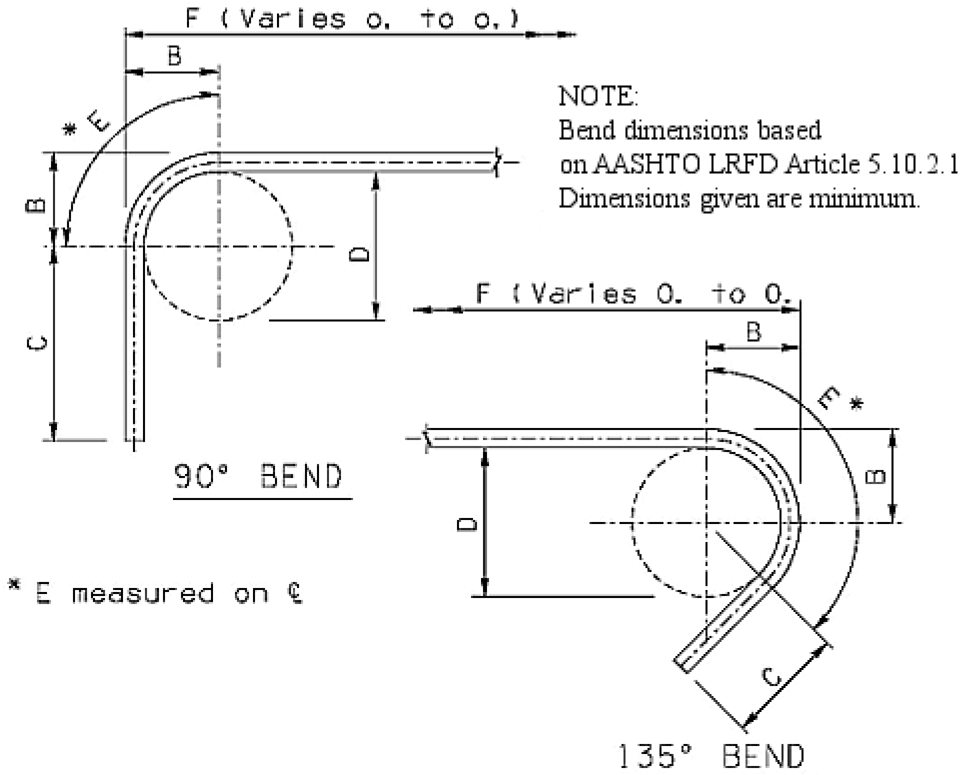
BEND DIMENSIONS (inches)										
BAR No.	BAR Dia.	D	C	B	A	E			TOTAL BAR LENGTH ADD TO F	
									1 HOOK	2 HOOKS
#3	.375	2 ¼	2½	1 ½	4	4 1/8			5 1/8	10 ¼
#4	.500	3	2½	2	4 ½	5 ½			6	12
#5	.625	3 ¾	2½	2 ½	5	6 7/8			6 7/8	13 ¾
#6	.750	4 ½	3	3	6	8 ¼			8 ¼	16 ½
#7	.875	5 ¼	3½	3 ½	7	9 5/8			9 5/8	19 ¼
#8	1.000	6	4	4	8	11			11	22
#9	1.128	9	4 ½	5 5/8	10 1/8	15 7/8			14 ¾	29 ½
#10	1.270	10 1/8	5 1/8	6 3/8	11 ½	17 7/8			16 5/8	33 ¼
#11	1.410	11 ¼	5 5/8	7	12 5/8	19 7/8			18 ½	37
#14	1.693	17	6 ¾	10 ¼	17	29 3/8			25 7/8	51 ¾
#18	2.257	22 5/8	9	13 5/8	22 5/8	39 1/8			34 ½	69

90^{o} STD. Hook Dimensions for Longitudinal Reinforcement



BEND DIMENSIONS (inches)										
BAR No.	BAR Dia.	D	C	B	A	E			TOTAL BAR LENGTH ADD TO F	
									1 HOOK	2 HOOKS
#3	.375	2 ¼	4 ½	1 ½	6	2			5	10
#4	.500	3	6	2	8	2 ¾			6 ¾	13 ½
#5	.625	3 ¾	7 ½	2 ½	10	3 ½			8 ½	17
#6	.750	4 ½	9	3	12	4 1/8			10 1/8	20 ¼
#7	.875	5 ¼	10 ½	3 ½	14	4 ¾			11 ¾	23 ½
#8	1.000	6	12	4	16	5 ½			13 ½	27
#9	1.128	9	13 ½	5 5/8	19 1/8	8			15 7/8	31 ¾
#10	1.270	10 1/8	15 ¼	6 3/8	21 5/8	9			17 7/8	35 ¾
#11	1.410	11 ¼	16 7/8	7	23 7/8	10			19 7/8	39 ¾
#14	1.693	16 7/8	20 3/8	10 1/8	30 ½	14 5/8			24 7/8	49 ¾
#18	2.257	22 5/8	27 1/8	13 5/8	40 ¾	19 ½			33	66

Standard Hook Dimensions for Transverse Reinforcement



BEND DIMENSIONS (inches)											
BAR No.	BAR Dia.	D	C 90°	C 135°	B	E 90°	E 135°	TOTAL BAR LENGTH ADD T0 F			
								1-90°	2-90°	1-135°	2-135°
#3	.375	1 ½	2 ¼	2 ¼	1 1/8	1 ½	2 ¼	2 5/8	5 ¼	3 3/8	6 ¾
#4	.500	2	3	3	1 ½	2	3	3 ½	7	4 ½	9
#5	.625	2 ½	3 ¾	3 ¾	1 7/8	2 ½	3 5/8	4 3/8	8 ¾	5 ½	11
#6	.750	4 ½	9	4 ½	3	4 1/8	6 ¼	10 1/8	20 ¼	7 ¾	15 ½
#7	.875	5 ¼	10 ½	5 ¼	3 ½	4 ¾	7 ¼	11 ¾	23 ½	9	18
#8	1.000	6	12	6	4	5 ½	8 ¼	13 ½	27	10 ¼	20 ½

In addition to the above tables, development of standard hooks in tension shall follow AASHTO LRFD Article 5.11.2.4.

5.10.3 Spacing of Reinforcement

5.10.3.3 Minimum Spacing of Prestressing Tendons and Ducts

5.10.3.3.1 Pretensioning Strand

Center-to-center spacing of strands shall be 2 inches. The use of bundled pretensioning strands shall not be allowed.

5.10.3.3.2 Post-Tensioning Ducts Not Curved in the Horizontal Plane

The clear distance between straight post-tensioning ducts in the vertical direction shall not be less than 1 inch and the clear distance in the horizontal direction shall follow the AASHTO LRFD specifications.

The use of bundled ducts shall not be allowed. In post-tensioned box girder bridges, ducts shall be arranged in vertical alignments only.

5.10.6 Transverse Reinforcement for Compression Members

5.10.6.2 Spirals

Welded wire fabric shall not be used as spirals. Welded splices shall meet the welding criteria specified in AASHTO / AWS D1.5 current edition and ADOT Standard Specifications for Road and Bridge Construction.

5.10.8 Shrinkage and Temperature Reinforcement

The permanent prestress of 0.11ksi, specified in the AASHTO LRFD Commentary 5.10.8, should not be added to that required for the strength or service limit state evaluations. It is the minimum requirement for shrinkage and temperature crack control.

5.10.9 Post-Tensioned Anchorage Zones

5.10.9.3 Design of the General Zone

For design of the general anchorage zones, any of the following design methods, conforming to the requirements of AASHTO LRFD Article 5.10.9.3.2, may be used:

Equilibrium-based inelastic models, generally termed as “strut-and-tie models”

Refined elastic stress analyses as specified in Section 4 and AASHTO LRFD Article 5.10.9.5

Other approximate methods Specified in AASHTO LRFD Article 5.10.9.6

5.10.9.3.3 Special Anchorage Devices

Where special anchorage devices that do not satisfy the requirements of AASHTO LRFD Article 5.10.9.7.2 are to be used, those devices shall be tested for compliance with the requirements of AASHTO LRFD Article 5.10.9.3.3.
ADOT Bridge Group will waive the testing requirements for the special anchorage devices provided that these anchorage systems have been tested and approved for use by California Department of Transportation (Caltrans). All documentation including the test results and the acceptance certificate from Caltrans must be provided by the vendor as part of Post-Tensioned

shop drawing submittal for the special anchorage devices. Reinforcements provided must be the same as was used by Caltrans during testing.

Design engineers are encouraged to communicate with the Post-Tension supplier in the design phase to get the recommendations for the minimum cover, spacing, and edge distances for an anchorage device that may be used. This will facilitate the design of the local and the general zones. Design engineers must show dimensions of general anchor zones in the contract plans. The local anchor zone shall be displayed as a schematic in the contract plans. Post-Tension supplier must show the local anchorage zone reinforcement to be supplied as a part of any proprietary Post-Tension system (special anchorage device). Any adjustments to the general anchorage zone tensile reinforcement due to change in the local zone dimensions shall be considered as part of the shop drawing approval process.

Anchorage Zones

A 4″x 4″ grid of #4 reinforcing behind the anchorage plate shall be used and detailed on the plans. When an anchorage device requires spiral, supplemental, or both, these approved reinforcements shall be in addition to the #4 grid. When a spiral on the end anchorage of a tendon conflicts with the grid system, the reinforcements in the grid may be re-spaced or cut as required.

C-shaped reinforcing consisting of #6 @ 4″ with 3′-0″ tails shall be placed along the exterior face of exterior web for the length of the diaphragm to aid in resisting bursting stresses.

5.11 DEVELOPMENT AND SPLICES OF REINFORCEMENT

5.11.2.1 Deformed Bars and Deformed Wire in Tension

5.11.2.1.1 Tension Development Length

The following table in addition to the following article of these guidelines shall be used for determining development length for deformed bars.

Basic Development Length For Deformed Bars in Tension (inch) is based on AASHTO LRFD Article 5.11.2.1.1

Bar Size (f_y = 60 ksi)	**f'_c=3.0 ksi**	**f'_c=3.5 ksi**	**f'_c=4.5 ksi**	**f'_c=5.0 ksi**	**f'_c=6.0 ksi**
#3	12.0*	12.0*	12.0*	12.0*	12.0*
#4	12.0	12.0	12.0	12.0	12.0
#5	15.0	15.0	15.0	15.0	15.0
#6	19.3	18.0	18.0	18.0	18.0
#7	26.3	24.3	21.5	21.0	21.0
#8	34.6	32.0	28.3	26.8	24.5
#9	43.8	40.6	35.8	33.9	31.0
#10	55.6	51.5	45.4	43.1	39.3
#11	68.4	63.3	55.8	52.9	48.3
#14	93.6	86.6	76.4	72.5	66.2
#18	121.3	112.3	99.0	94.0	85.8

* The calculated development length, l_d, for # 3 bar is 9″, however l_d shall not be less than 12" except in computation of lap splices by AASHTO LRFD Article 5.11.5.3.1 and in calculation for closed stirrup requirement in AASHTO LRFD Article 5.11.2.6.4.

5.11.2.1.2 Modification Factors That Increase l_d

The basic development length, l_d, shall be multiplied by the following factors, as applicable.

Modification Factors	
Top horizontal reinforcement so placed that more than 12 inches of fresh concrete is cast below the reinforcement.	1.4
Epoxy coated reinforcements with cover ▪ Less than 3 reinforcing bar diameter (d_b) cover or 6 d_b clear spacing between reinforcements ……….. ▪ All other cases ………………………………….	 1.5* 1.2
Lateral spacing ≥ 6” with minimum 3” clear cover in the direction of the spacing	0.8
Excess reinforcement - $(As_{required}) \leq (As_{provided})$	$\frac{(As_{required})}{(As_{provided})}$
Enclosed within spirals (≥1/4” Φ and ≤ 4” pitch)	0.75

* The product obtained when combining the factor for top reinforcement with the applicable factor for epoxy coated reinforcement need not be taken to be greater than 1.7

5.12 DURABILITY

Concrete structures shall be designed to provide protection of the reinforcing and prestressing steel against corrosion throughout the life of the structure. Protection measures for durability shall satisfy the requirements specified in AASHTO LRFD Article 2.5.2.1. All new bridge construction located above an elevation of 4000 feet, or for areas where de-icing chemicals are used, deck slabs, barriers, anchor slabs and approach slabs reinforcing as well as portions of reinforcement projecting into the deck slabs shall be epoxy coated.

Use of High Performance Concrete (HPC) could be considered as a measure of enhancing concrete durability if approved by ADOT Bridge Group.

5.13 SPECIFIC MEMBERS

5.13.2.2 Diaphragms

For post-tensioned box girder bridges, a single 9-inch thick intermediate diaphragm shall be placed at the midspan. Special consideration for additional diaphragms shall be given to box girders with large skews, curved boxes and boxes over seven feet in depth. Diaphragms shall be placed parallel to abutments and piers for skews less than or equal to 20 degrees. Diaphragms shall be placed normal to girders and staggered for skews over 20 degrees. All diaphragms shall be cast integral with girder webs to add lateral stability to the forming system.

For prestressed precast I-Girder bridges, a single 9-inch thick intermediate diaphragm shall be placed at the midspan for all spans over 40 feet. For skews less than or equal to 20 degrees, the diaphragm shall be placed parallel to the skew. For skews greater than 20 degrees, the diaphragms shall be staggered and placed normal to the girder.

5.14 PROVISIONS FOR STRUCTURE TYPES

5.14.1 Beams and Girders

5.14.1.2 Precast Beams

Precast prestressed girders shall be designed as simply supported beams using composite section properties for dead and transient loads. The superstructure shall be constructed continuous over the intermediate supports and designed for transient and composite dead load. The design should include the effects of shrinkage and creep for all strength limit states. Additional non-prestressed reinforcement shall be provided in the deck slab to account for continuity over the intermediate supports. The design shall be based on the strength of concrete of the closure pour. The additional reinforcement shall meet the requirements of AASHTO LRFD Article 5.11.1.2.3.

5.14.1.2.4 Detail Design

Differential shrinkage shall be considered in the design.

5.14.1.4 Bridges Composed of Simple Span Precast Girders Made Continuous

The design engineer shall follow AASHTO LRFD Article 5.14.1.2.7, third edition, 2004. The current AASHTO LRFD Article has been rescinded.

The positive moment connection may be designed using the methods described in the PCA publication “Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders”, August 1969. In determining the positive restraint moment using the referenced PCA publication, the duration between casting of the girders and deck closure may be taken as 30 days.

The development length of the strands may be based on criteria contained in Report No. FHWA-RD-77-14, “End Connections of Pretensioned I-Beam Bridges” November 1974. The equations from the report are reproduced below for convenience.

$$L_e = 0.228 f_{ps} + 8.25, \qquad \text{where } L_{pb} \le 8.25''$$

$$L_e = 0.228\left[f_{ps} - \frac{L_{pb} - 8.25}{0.472}\right] + L_{pb}, \qquad \text{where } L_{pb} \ge 8.25''$$

where:

L_e = embedment length of strand (inch)
L_{pb} = prebend length of embedded strand (inch)
f_{ps} = allowable stress per strand (inch)

The above relationship may be rewritten as

$$f_{ps} = \frac{L_e - 8.25}{0.228} \leq 150\text{ksi} \qquad \text{where } L_{pb} \leq 8.25''$$

The required number of strands to be extended may be determined using the following equation:

$$A_{ps(req'd)} = \frac{M^+ - A_s f_y (jd_{ps} + d - d_{ps})}{f_{ps} jd_{ps}}$$

where:

M^+ = Positive moment evaluated from PCA publication
A_s = Area of diaphragm ties
d = distance from extreme compressive fiber to the centroid of the diaphragm
d_{ps} = distance from extreme compression fiber to the centroid of the strands
jd_{ps} = internal moment arm, may be assumed as = $0.94d_{ps}$

Once the number of strands and embedment length have been determined, the section should be checked for ultimate capacity following the procedure presented in the referenced FHWA report.

$$Mu = \Phi \left[A_{ps} f_{pu} \left(d_{ps} - \frac{a}{2} \right) + A_s f_y \left(d - \frac{a}{2} \right) \right]$$

$$f_{pu} = \frac{L_e - 8.25}{0.163}$$

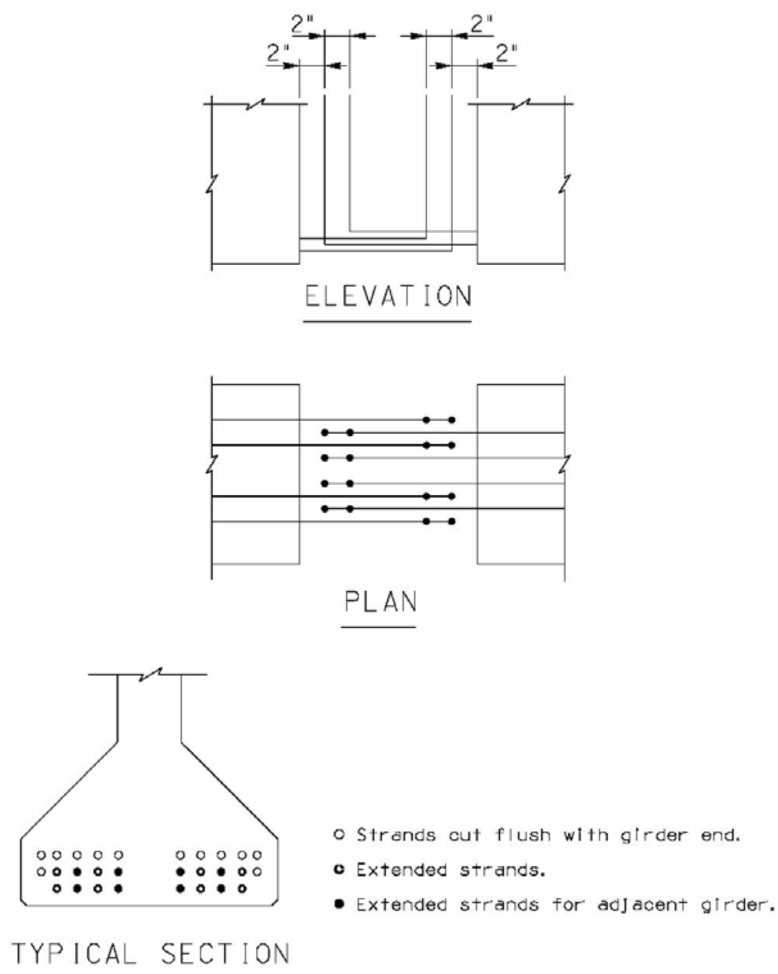
$$a = \frac{A_{ps} f_{pu} + A_s f_y}{0.85 f_e b}$$

where:

a = depth of compression stress block
b = width of member
f_{pu} = stress of strands at general slip
Φ = workmanship factor, 0.90 for flexure

In determining the number and pattern of strands extended, preference shall be given to limiting the number of strands by increasing the extension length and alternating the pattern to increase constructability. Refer to the figure below for strand pattern at girder end.

STRAND PATTERN AT GIRDER END.



Method of Analysis for Precast I Girders

Section properties shall be based on transformed area of bonded prestressing strands for precast prestressed members.

The girders shall be designed as simply supported beams for dead load and live load plus impact.

Girders shall be designed using the pretensioning method only. Post-tensioned alternates shall be used only for large projects when approved by ADOT Bridge Group.

Use of masked strands for debonding shall not be allowed.

The location of the harped points of the strand should be as required by design with the preferable locations being near the 1/10 of the span as measured from the midspan of the girder.

AASHTO Type V modified and Type VI modified I-girders should be used in place of Type V and Type VI girders whenever possible.

The theoretical build-up depth shall be ignored for calculation of composite section properties.

5.14.1.5 Cast-in-Place Girders and Box and T-Beams

5.14.1.5.1 Flange and Web Thickness

5.14.1.5.1a Top Flange

Minimum top flange thickness shall be 8″, refer to Section 9 of these guidelines.

5.14.1.5.1b Bottom Flange

Minimum bottom flange thickness shall not be less than 6 inches.

5.14.1.5.1c Web

Web thickness shall not be less than 12 inches (measured normal to girder for sloping exterior webs). Exterior girders webs shall be flared to a minimum thickness of 18 inches at the abutments. The flare length shall be 16 feet from the face of the abutment diaphragm. Interior webs shall be constructed vertical. A 4″x 4″ fillet shall be used at the tops of interior webs but is not required at the bases.

For girders over 8.0 feet in depth, AASHTO LRFD C5.14.1.5.1c shall be followed.

Method of Analysis for Cast-In-Place Box Girder

Section properties shall be based on gross area of members for cast-in-place post-tensioned members.

The bottom slab, in the vicinity of the intermediate support, may be flared to increase its thickness at the face of the support when the required concrete strength exceeds 4.5 ksi. When thickened, the bottom slab thickness should be increased by a minimum of 50 percent. The length of the flare should be at least one-tenth of the span length (measured from the center of the support) unless design computations indicate that a longer flare is required.

Section properties at the face of the support should be used throughout the support; i.e., the solid cap properties should not be included in the model.

Negative moments should be reduced to reflect the effect of the width of the integral support.

The combination of dead load and prestress forces should not produce any tension in the extreme fibers of the superstructure.

Cast-in-Place multicell concrete box girder bridges shall be designed as one unit for the entire cross-section of the superstructure. Such cross-sections shall be designed for live load distribution factors specified in AASHTO LRFD Articles 4.6.2.2.2 and 4.6.2.2.3 for interior girders, multiplied by the number of girders, i.e., webs.

For box girders with severe sloping webs or boxes over 7 feet deep, transverse flange forces induced by laterally inclined longitudinal post-tensioning shall be considered in the design.

Single span structures should be jacked from one end only. Symmetrical two span structures may be jacked from one end or from both ends. Unsymmetrical bridges should be jacked from the long end only or from both ends as required by the design. Three span or longer structures should be jacked from both ends.

Several prestressing systems should be checked to verify that the eccentricity and anchorage details are acceptable. In determining the center of gravity of the strands, the difference between the center of gravity of the strands and the center of the ducts, shall be considered. For structures over 400 feet in length, in determining the center of gravity of the strands, the diameter of the ducts should be oversized by ½ inch to allow for ease of pulling the strands.

For horizontally curved bridges, special care shall be taken in detailing stirrups and duct ties. Loss of prestress due to friction should be based on both vertical and horizontal curvatures. In designing for horizontal curvature, the exterior web with smallest radius shall be used. A variation of prestressed force not to exceed 5% per web shall be allowed provided that total jacking force remains the same as calculated in design. The design engineer may read and implement recommended design criteria included in the article "The Cause of Cracking in Post-Tensioned Concrete Bridges and Retrofit Procedures", by Walter Podolny, published in the PCI Journal, Vol. 30, No. 2, March-April 1985.

5.14.4 Slab Superstructures

5.14.4.3 Precast Deck Bridges

Prestressed Precast Box Beams

End Blocks
End blocks 18 inches long shall be provided at each end and sufficient mild reinforcing shall be provided in the end blocks to resist the tensile forces due to the prestressing loads.

Diaphragms
Diaphragms, cast within the beam, shall be provided at the midspan for spans up to 50 feet, at the third points for spans from 50 to 75 feet and at quarter points for spans over 75 feet.

Lateral Ties
One lateral tie shall be provided through each diaphragm located at the mid-depth of the section. However, for 39-inch or deeper sections, when adjacent units are tied in pairs for skewed bridges, in lieu of continuous ties, two ties shall be provided, located at the third points of the section depth.
Each tie shall consist of a 1½-inch diameter mild steel bar tensioned to 30,000 pounds. Tension in the 1½-inch diameter mild steel should be applied by the turn of nut method. The design engineer should determine the number of turns of the nut required to achieve the 30,000 pounds force. This value should be shown on the plans.
ASTM A36 steel bars for the tie normally come in 20-foot lengths. The final total length of the tie should be made using threaded couplers; not welded splices. When couplers are used, the hole through the diaphragm should be increased from the normal 2½ inches to 4 inches diameter to accommodate the couplers.
Adequate means shall be used to ensure that the ties are adequately protected from corrosion. The rod, nut and bearing plate shall be galvanized in accordance with ASTM A153 (AASHTO M-232).

Shear Keys
After shear keys have been filled with an approved non-shrink mortar and reached a minimum strength of 5,000 psi, lateral ties shall be placed and tightened.

Prestressed Precast Voided Slabs

End Block
End blocks should be 15 inches long with sufficient mild reinforcing provided to resist the tensile forces due to concentrated prestressing loads.

Diaphragms
Diaphragms shall be cast within the slab at midspan for spans up to 40 feet and at third points for spans over 40 feet.

Barriers
Barriers shall have a ½ inch open joint at the midspan to prevent the barrier from acting as an edge beam and causing long-term differential deflection of the exterior beam.

5.14.4.3.3 Shear-Flexure Transfer Joints

Lateral Ties
One lateral tie shall be provided through each diaphragm located at the mid-depth of the section. Each tie shall consist of a 1½-inch diameter mild steel bar tensioned to 30,000 pounds. Tension in the 1½-inch diameter mild steel should be applied by the turn of nut method. The design engineer should determine the number of turns of the nut required to achieve the 30,000 pounds force. This value should be shown on the plans.
ASTM A36 steel bars for the tie normally come in 20-foot lengths. The final total length of the tie should be made using threaded couplers; not welded splices. When couplers are used, the hole

through the diaphragm should be increased from the normal 2½ inches to 4 inches diameter to accommodate the couplers.

Adequate means shall be used to ensure that the ties are adequately protected from corrosion. The rod, nut and bearing plate shall be galvanized in accordance with ASTM A153 (AASHTO M-232).